CONCRETE
AND
CONSTRUCTIONAL ENGINEERING

INCLUDING PRESTRESSED CONCRETE

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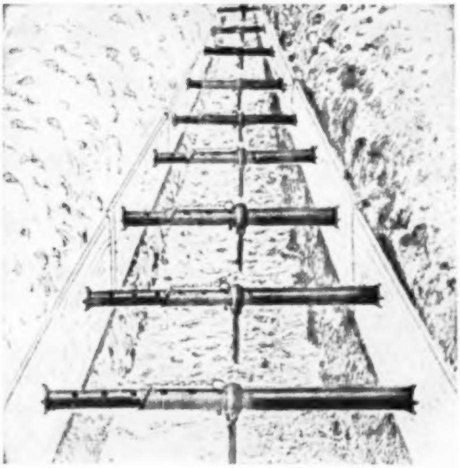

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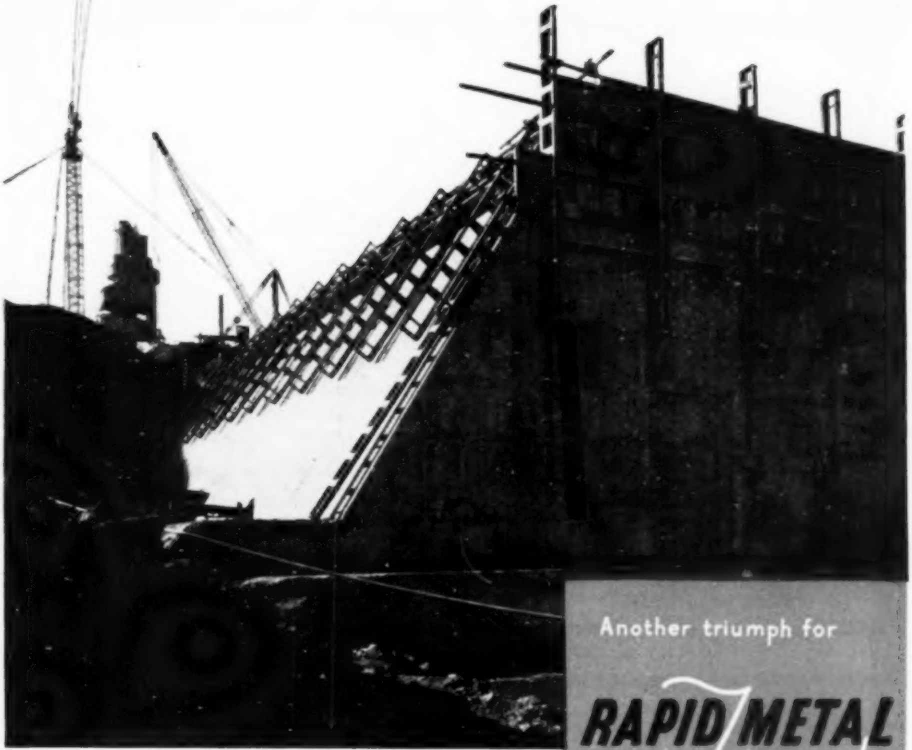

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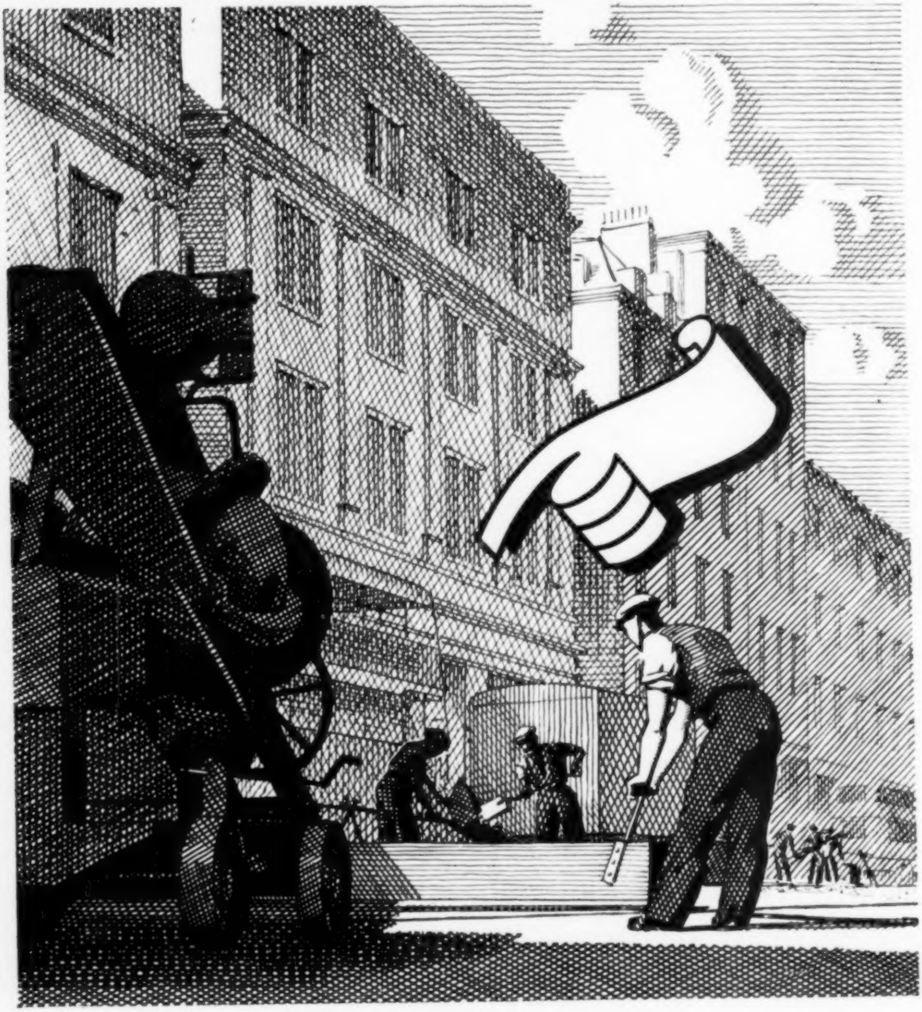

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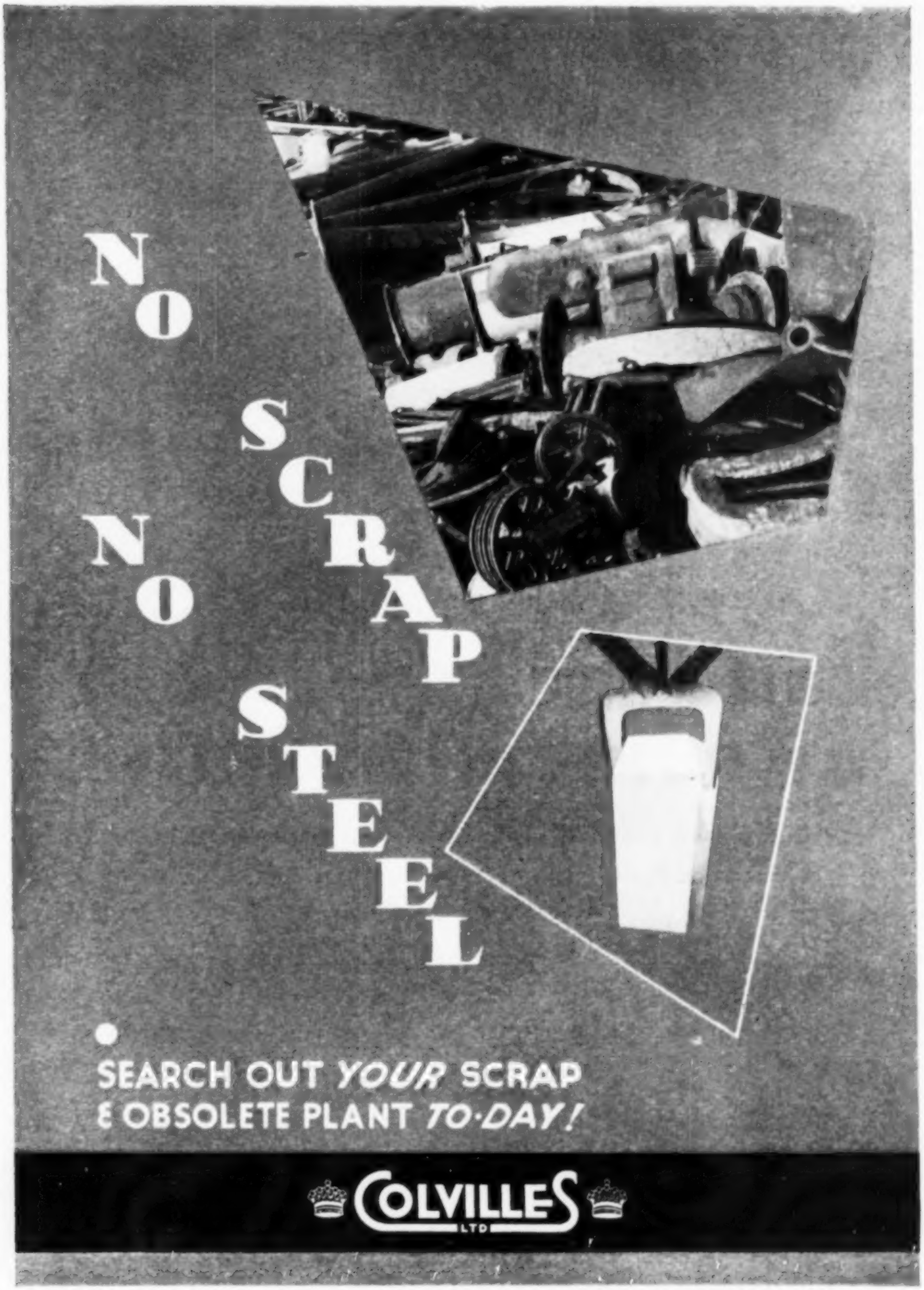
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Volume XLVIII, No. 5. LONDON, MAY, 1953.

EDITORIAL NOTES

Building Research.

AT a conference on research held by the National Federation of Building Trades Employers recently, the secretary of the Department of Scientific and Industrial Research said he did not believe that the wider and speedier application of science in the building industry could be brought about by an extension or intensification of effort on the present lines. He also expressed the views that the industry generally did not take sufficient interest in science and research, and that " builders would be far readier to listen to those within their own trade than to outsiders like scientists ". The Federation was urged to set up its own research groups for the purpose of sponsoring and organising investigations, for receiving " information in bulk " from scientists, and for providing " wider and more systematic channels " of co-operation with the research worker.

It would be hardly surprising if builders were sceptical of the value to them of official research, for in concrete and reinforced concrete at any rate it is difficult to name any recent improvement in materials, processes, or machinery used in the industry which has originated from a British scientist, a British official research laboratory, or indeed anywhere in Britain. It is lamentable that so many of the developments in concrete and reinforced concrete have come from abroad. The fact that reinforced concrete was invented in France is not a good reason why none of the developments during the past hundred years originated in this country, particularly during the past thirty years when such large sums have been spent here on research. During this period of intensified research the British contribution to the development of concrete and reinforced concrete has been negligible.

For example, among the newer lightweight aggregates, foamed slag was used first in Austria and Germany, and expanded slate and vermiculite in the United States. Aerated and " gas " concretes were mainly introduced into this country when German patents and processes were revealed after the war. Air-entraining admixtures and " vacuum " concrete were developed in America many years ago, and were only recently available here. " No-fines " concrete was used in Holland forty or more years ago. Sawdust-concrete and wood-wool-cement slabs were first used in Central Europe. Types of rapid-hardening or high-early-strength cements appear to have been made on the Continent early in the century, and in this country nearly twenty years later. High-alumina and expansive cements were French discoveries, and blastfurnace cements were first made in Germany. Among the newer types of reinforcement, twin-twisted high-tensile

bars were first used in Germany and indented bars in France, Germany, and the United States.

In the making of concrete and building in concrete we have also largely depended on ideas and improvements from abroad. Weigh-batching mixing plant came from America, as also did chuting towers and the first systems of steel shuttering. Electric vibrating machines of the shutter and immersion types originated on the Continent. Tower cranes are imported from France. The concrete pump and pipeline are of Continental origin. Modern earth-moving equipment was introduced from the United States for the construction of runways during the war. The machine which spreads, tamps, and finishes concrete for roads was produced in Germany in the 1930's. The delivery of loose cement in special vehicles originated on the Continent, and the cement-pump in the U.S.A. The use of paper valve-bags instead of jute sacks for cement was an American idea. Central concrete-mixing plants and vehicles for mixing concrete in transit were common in the United States and elsewhere before they were introduced here. Soil-cement roads were first built in the United States and elsewhere. The chemical consolidation of soil is a Continental invention. The centrifugal process of making concrete pipes and poles came from the Continent and Australia, and continuous roofing-tile machines and many other machines for making concrete products from the Continent.

Although work done in this country on the creep of concrete is of value in the design of prestressed concrete, the major practical applications of prestressed concrete were due entirely to Continental engineers; it must, however, be mentioned as a notable exception that the latest development in the manufacture and use of post-tensioned high-tensile bars is the invention of British engineers. The "shell" roof originated in Germany. In Great Britain the possibilities of facilitating construction by precasting structural members are now realised, but progress has been slow since, in the 1920's, thirty two-story houses were built at Cambridge by a Continental "tilt-up" method in which complete walls were cast on the ground and hoisted into position by a crane. Some of the precast beams made in Great Britain are based on Continental or American designs. It is a revealing experience to see in foreign publications illustrations of the remarkable sizes and shapes of precast members used abroad, particularly in Italy, Czechoslovakia, and Central and South America.

It is depressing to find that a nation that at one time claimed to lead the world in inventiveness has contributed so little to progress in concrete and reinforced concrete, and it would not be surprising if the industry looked upon scientists as "outsiders", for it is hardly to be expected that inventions and ideas such as those mentioned would come from scientists working in a laboratory. It has often been said that the building industry is backward in adopting new ideas, but we have yet to meet an engineer or builder who would not do so when it was proved to his satisfaction that a new material or process or machine would be advantageous to him. But such ideas mostly come from manufacturers and builders actively employed in their trades, and not from official laboratories whose work might be better confined to "fundamental" research of the kind now being carried out by the Department of Scientific and Industrial Research on the constitution of Portland cement, and on such matters as the fire-resistance and durability of materials and the testing of full-size structures.

Prestressed Precast Concrete Footbridges.

By N. A. DEWS.

In this journal for August, 1952, the writer gave a design for a footbridge consisting of prestressed precast concrete units for use on remote sites. If, however, the site is within a reasonable distance from a hard road the alternative design shown in *Fig.* 1 can be quickly erected and is low in first cost and maintenance cost. The bridge comprises two precast prestressed beams connected by small in-situ diaphragms which also serve as supports for the precast prestressed deck.

If many bridges are to be provided by one authority, there will almost certainly be several that can be grouped into spans of standard length, so that adherence to the minimum span would be unnecessary. The beams would be factory-made with bonded wires, and designed so that no special precautions in handling or transporting are necessary.

The method of determining the prestressing force is similar to the method suggested in the previous article but with a slight modification of the stresses in the top face. During transport or while being placed in position the beam may possibly be slung from points other than the ends, or it may be rolled into position; the most severe case will be when it is supported at the centre, thereby producing a reversal of the usual stresses due to bending. If tensile stresses are not permitted, and this is advisable for these small beams, precompression must be provided in the top face equal to or greater than the tensile stress c_t'' due to bending; the eccentricity of the stressing force must then be within the core of the beam, that is less than $\frac{r^2}{y_t}$. For this condition of reversed bending moment the beam could be handled either before or after all the losses of prestress have occurred, and the lower limit is

$$\frac{nP}{A}\left(1 - \frac{ey_t}{r^2}\right) - c_t'' = 0 \qquad . \quad . \quad . \quad . \quad . \quad (1a)$$

For the upper limit at the top face, that is with the beam in position and all the loads acting, the permissible compressive stress c_a must not be exceeded; this condition can occur before losses of prestress have taken place, and the limit is

$$\frac{P}{A}\left(1 - \frac{ey_t}{r^2}\right) + c_t + c_t' = c_a \qquad . \quad . \quad . \quad . \quad . \quad (2a)$$

The two limits for the bottom face are as given in the previous article. The ordinates for $1000 . A \div P$ where $e = 0$ are: (1*a*), $\frac{1000n}{c_t''}$; (2*a*), $\frac{1000}{c_a - c_t - c_t'}$; (3), $\frac{1000}{c_a + c_b}$; and (4), $\frac{1000n}{c_b + c_b'}$.

With $e = \frac{r^2}{y_t}$ in cases 1*a* and 2*a* and $e = -\frac{r^2}{y_b}$ in cases 3 and 4 as before, a diagram similar to *Fig.* 2 results, giving a kern in which are acceptable values for P and e for the required conditions.

The properties of the section in *Fig.* 1 are as follows (in inch units): $A = 68$,

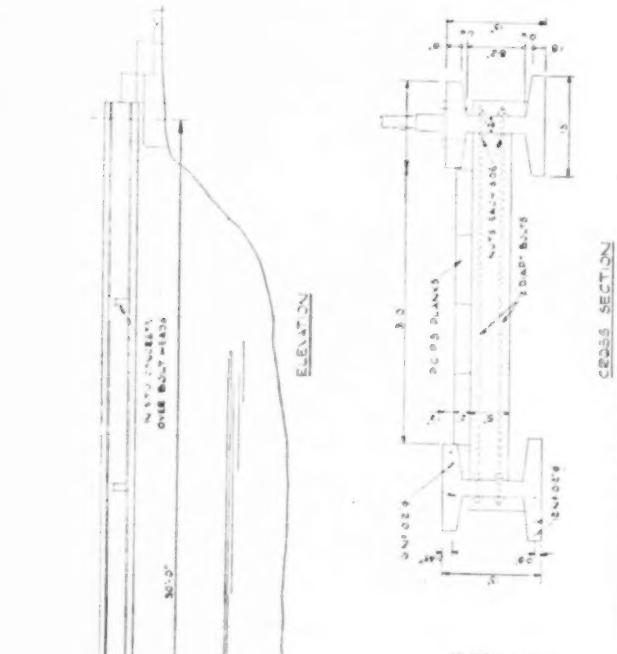


Fig. 1.—Design for a Prestressed Concrete Footbridge.

$y_t = 6{\cdot}85$, $y_b = 6{\cdot}15$, $I = 1620$, $r^2 = 23{\cdot}82$, $\frac{r^2}{y_t} = 3{\cdot}48$, $\frac{r^2}{y_b} = 3{\cdot}87$; weight per foot $= 72$ lb.

When the beam 31 ft. long is slung or supported at the centre: $M = \frac{1}{2}15{\cdot}5^2 \times 72 \times 12 = 103{,}790$ in.-lb.; $c_t'' = \frac{103{,}790 \times 6{\cdot}85}{1620} = 439$ lb. per square inch (tension).

With the beam in position: $M = \frac{1}{8}72 \times 30^2 \times 12 = 97{,}200$ in.-lb.

$$c_t = \frac{97{,}200 \times 6{\cdot}85}{1620} = 411 \text{ lb. per square inch (compression).}$$

$c_b = \frac{97{,}200 \times 6{\cdot}15}{1620} = 369$ lb. per square inch (tension). The loads added after the beam is placed are (in lb. per foot): Handrail 8, diaphragms 4, deck 38; total 50. The live load at 80 lb. per square foot is 120, and the bending moment due to these added loads is $\frac{1}{8}170 \times 30^2 \times 12 = 229{,}500$ in.-lb. $c_t' = 970$ lb. per square inch (compression) and $c_b' = 870$ lb. per square inch (tension).

The ordinates for $e = 0$ in *Fig.* 2 are:

$$(1a), \frac{850}{439} = 1{\cdot}94; \quad (2a), \frac{1000}{2000 - 411 - 971} = 1{\cdot}62;$$

(3), $\frac{1000}{2000 + 369} = 0{\cdot}422$; and (4), $\frac{850}{369 + 870} = 0{\cdot}685$. It is seen that e is between 1·4 and 2·3 with $1000.A \div P$ between 1·02 and 0·65; with $A = 68$,

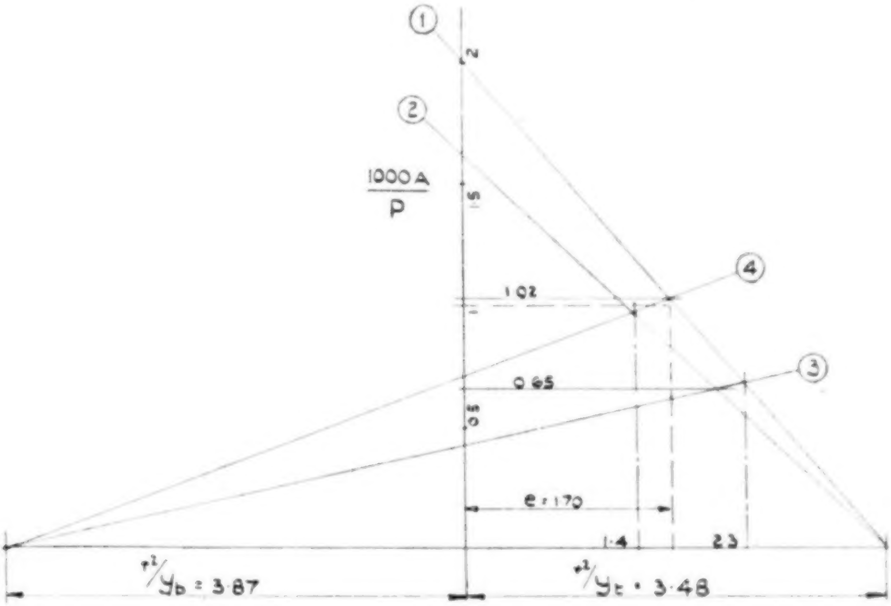


Fig. 2.

P is 66,700 to 104,600 lb. Using 18 wires of 0·2-in. diameter stressed to 120,000 lb. per square inch a force of 67,900 lb. is provided; with this force $1000A \div P$ is nearly 1, indicating an eccentricity of 1·7 in. (it should be noted that extreme accuracy is not necessary in these diagrams as the final stresses for the values chosen for P and e are derived mathematically).

The stresses resulting from the force of 67,900 lb. are

$$p_t = \frac{67{,}900}{68}\left(1 - \frac{1{\cdot}7}{3{\cdot}48}\right) = 509 \text{ lb. per square inch (compression).}$$

$$p_b = \frac{67{,}900}{68}\left(1 + \frac{1{\cdot}7}{3{\cdot}87}\right) = 1438 \text{ lb. per square inch (compression).}$$

Allowing 15 per cent. loss of prestressing force for creep, shrinkage, etc., these stresses become in course of time 446 lb. and 1258 lb. per square inch respectively. The combined stresses are: Top face—(1*a*), $446 - 439 = 7$ lb. per square inch. (2*a*), $509 + 411 + 970 = 1890$ lb. per square inch. Bottom face—(3), $1438 - 369 = 1069$ lb. per square inch; (4), $1258 - 369 - 870 = 19$ lb. per square inch. These stresses are all compressive, and as they are less than the permitted value of 2000 lb. per square inch the conditions are satisfied. Twelve wires are placed 0·9 in. from the bottom face and the remaining six wires are placed a distance from the top equal to $6{\cdot}85 - \dfrac{12 \times 5{\cdot}25 - 18 \times 1{\cdot}7}{6} = 1{\cdot}25$ in.

Considering the condition without live load, the moment due to the total dead load of $72 + 50$ lb. per foot is 164,700 in.-lb., producing stresses of 696 lb. and 625 lb. per square inch, and the combined stresses are then: Top: Due to bending, 696; initial prestress, 509; total, 1205 lb. per square inch (compressive). Bottom: Due to bending, -625; reduced prestress, 1258; total, 633 lb. per square inch (compressive).

Should the beam be off-loaded upside down, the prestress would remain the same but the distances to the neutral axis would be reversed; in this case no tensile stresses would occur, nor would the compressive stress be exceeded if the beam were supported at the centre. If the beam, still upside down, were supported at the ends the stresses c_t and c_b would be reversed and the condition would then be: Upper face (bottom of beam)—Due to bending, 369; initial prestress, 1438; total, 1807 lb. per square inch. Lower face (top of beam)—Due to bending, -411; reduced prestress, 446; total, 35 lb. per square inch. Lines for this or other conditions can be added to the diagram in *Fig.* 2, but the limiting cases can generally be reduced to four.

The 2-in. thick deck planks weigh about 20 lb. per foot; allowing 60 lb. per foot for live load on the 9-in. width, the bending moment on the span of 6 ft. is 4320 in.-lb., resulting in stresses of $\pm$ 720 lb. per square inch. A uniform compression of 750 lb. per square inch is produced by four wires. These planks have a small hole in each end for dowelling them to one another, and the end planks are concreted in with the in-situ concrete.

Extension of the Dorchester Hotel, London.

THE Dorchester Hotel in Park Lane, London, was built in 1930, and has recently been extended by the provision of further bedrooms on one of the wings facing Deanery Street. When the hotel was designed this portion comprised a garage in the basement and a ground-floor car park. No extensions were then contemplated above the level of the pavement. In the year 1938, however, two banqueting rooms were built above ground-floor level and a third room constructed as a mezzanine in the garage. The present alteration required one of the wings to be extended outwards and built up to the full height of the hotel (about 90 ft. above the level of the pavement), and it was necessary to provide the supports for the additional six floors without preventing the use of the existing rooms below, or interfering with the use of the garage.

The extension is about 60 ft. long from the end of the existing wing and is 40 ft. wide. The floors had to span this width without intermediate support. There were only two possible positions in the garage for the main columns where they would not obstruct the entrances to the banqueting rooms, and where there was sufficient space in the basement to enable the column-bases to spread the load on the ground at the stipulated safe pressure of 3 tons per square foot. Therefore the whole of the additional six floors are carried on two main columns at the eastern end with a support on the wall of the existing wing. *Fig.* 1 is a diagrammatic sketch by the architect of the new extension and its supports.

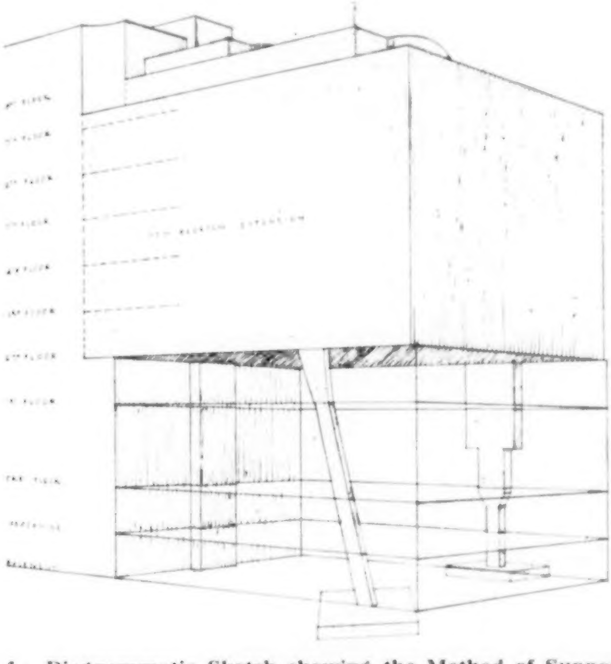


Fig. 1.—Diagrammatic Sketch showing the Method of Supporting the New Extension.

The construction of the lowest five new floors consists of cross beams 2 ft. deep spanning 40 ft., and connected by a solid reinforced concrete slab 4 in. thick. The ends of the cross-beams are carried on the side walls of the extension. These walls

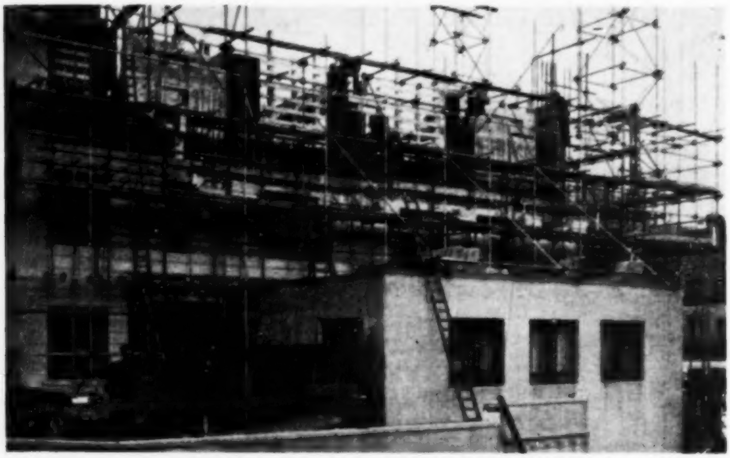


Fig. 2.—Work between Second and Third Floors.

Fig. 3.—Photograph taken Eleven Weeks after that in Fig. 2.

comprise a reinforced concrete core 5 in. thick with 2 in. of cork on the inner face and terrazzo slabs 2 in. thick on the outer face. This is the same form of construction as in the main hotel, except that in the main building the reinforced concrete core is 7 in. thick. The new side walls act as girders which span from the end wall of the existing wing for a distance of about 42 ft. to the main columns, and cantilever beyond them about 18 ft. to the end wall of the extension. The side walls were built two stories high and were designed in the first stage to carry two floors of the extension before the supports were removed. As each floor was added and the height of the wall increased, the tensile stress in the main bottom reinforcement also increased progressively but to a diminishing extent. In these side walls the presence of large window openings caused concentrations of high shear stresses.

The new side walls commenced at second-floor level, which was the level of the roof of the existing building, and provision was made to avoid the load from these walls bearing on the existing work by forming a space between the roof and the bottom of the wall which was subsequently filled with compressible expansion-joint material. The main column on the south was positioned at second-floor level in the manner described, but it was not possible for this column to be vertical as it would have obstructed the main aisle in the garage below. This column therefore slopes at an angle of about 1 in $4\frac{3}{4}$, thereby exerting a thrust on the existing hotel. On the north side the main column is not directly under the side wall, and a large beam was introduced at third-floor level to carry the load of 543 tons from the wall for a distance of about 1 ft. 1 in. to the centre of the new column extending a height of 55 ft. from the basement floor. To avoid eccentric loading on this column a hinge was introduced below the main transverse beam so as to ensure that all the load on the main column is truly concentric. The column is the shape shown in *Fig.* 1 to enable certain obstructions to be avoided and to preserve an existing window-opening. *Fig.* 4 shows the sloping column.

The existing roof at second-floor level was designed for normal roof loads and the stresses, when calculated for the floor-load plus the weight of partitions, proved to be somewhat excessive. Hangers were introduced on the line of the new corridor partition walls, and these were extended up to the new third-floor cross beams. These hangers were stressed by hydraulic jacks and the nuts then tightened. In this way an upward deflection of the existing second-floor beams was obtained and the extra load on the second floor is shared between that floor and the new third floor which was designed accordingly.

Fig. 4.—Part of Sloping Column.

At seventh-floor level the walls are set back on three sides of the extension, forming a penthouse of two stories. The walls are of similar construction to those below and are carried on cross-beams at seventh-floor level. The penthouses are to be used as residences and are provided with a roof-garden having two fish-pools. The eighth floor and roof are of reinforced concrete hollow construction; they are 14 in. deep and span 33 ft. with ribs at 2 ft. 6 in. centres and a top slab 2 in. thick.

The concrete throughout the work was a

1 : 1½ : 3 mixture and had an average compressive strength at 28 days of 5530 lb. per square inch. The design is in accordance with B.S. Code of Practice No. 114 (1948). The work was commenced in September, 1952, and the whole of the structural work was completed during the first week in February, 1953. In view of the very restricted site and the difficulties in unloading and storing materials and mixing the concrete, this work was an outstanding achievement in speed of construction in reinforced concrete. *Figs.* 2 and 3 illustrate the rapid progress made in the construction of the reinforced concrete work.

The architects are Messrs. Curtis Green, Son & Lloyd, the structural engineers are Considère Constructions, Ltd., and the contractors Sir Robert McAlpine & Sons, Ltd., all of whom were responsible for similar work in the main hotel.

Reconstruction of a Railway Station at Sheffield.

The roof over the main-line platforms of Victoria Station, Sheffield, is being rebuilt after the collapse in 1951 of four trusses at the western end of the station. The new work incorporates seventeen prestressed precast beams spanning across the tracks and supported by existing walls at a height of about 26 ft., and carrying over the platforms glazed steel awnings. The beams are at 25 ft. intervals and each weighs about 12½ tons. In addition to the awnings the beams are designed to carry overhead electrical equipment which will be required when the lines are electrified.

The beams are 87 ft. 9 in. long with a depth of 3 ft. 3 in. for a central length of 33 ft. reducing to 1 ft. 9 in. at the ends. The cross section is as *Fig.* 1 except for lengths of about 5 ft. at each end where the beam is rectangular. Each beam is prestressed by two straight cables each containing sixteen high-tensile wires of 0·276 in. diameter. The beams also contain twelve 0·276 in. untensioned wires, twisted in pairs, of which four are in the top flange and eight in the bottom flange. Between the flanges and at about 8 ft. centres are stiffening ribs, in the outer three of which at each end of the beam are cast sockets for the steel suspension rods of the awnings. The innermost suspension rod passes through a socket cast in the rectangular end-portions of the beams.

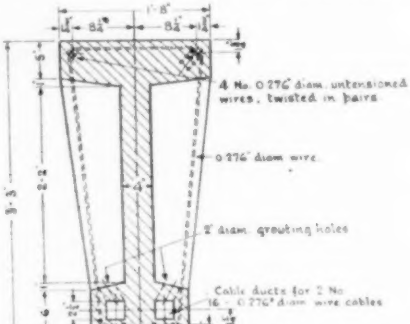


Fig. 1.—Cross Section of Beams.

The design of the beams, which are prestressed by the Magnel-Blaton system, was carried out under the supervision of Mr. J. I. Campbell, M.I.C.E., Civil Engineer of the Eastern Region of British Railways. These beams are being made and erected by Messrs. Wellerman Bros., Ltd., and the remainder of the work is being carried out by Messrs. Samuel Butler & Co., Ltd.

Gap-Graded Aggregates.

By J. D. O'KEEFFE, B.E.

An investigation has been made on the effects of omitting material between $\frac{3}{8}$ in. and $\frac{3}{16}$ in. from the continuous type-gradings Nos. 5 and 6 given by Professor H. N. Walsh for $\frac{3}{4}$-in. gravel aggregate [1]. Two cement contents (about 520 lb. and 610 lb. per cubic yard) were used, corresponding roughly to 1 : 2 : 4 and 1 : $1\frac{1}{2}$: 3 concretes. The material below the gap was sand; that above it was

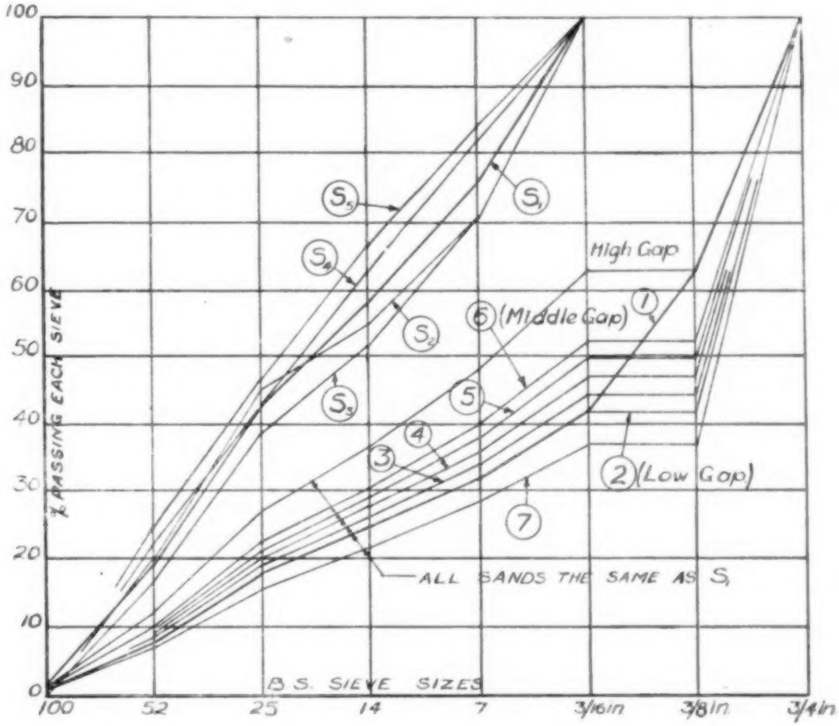


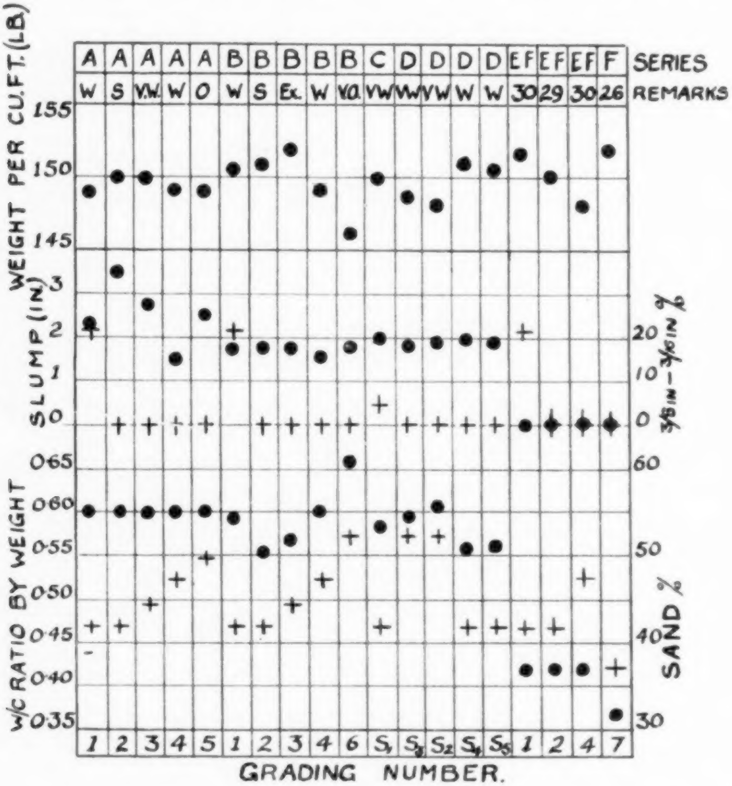
Fig. 1.—Gradings for Mixtures Containing 520 lb. of Cement per Cubic Yard.

coarse aggregate. The aggregate was rounded limestone gravel with a small proportion of crushed gravel. A gap of this nature can be obtained by using a single size ($\frac{3}{4}$-in. to $\frac{3}{8}$-in.) coarse aggregate. A near-gap, that is when only a small amount of material of the gap size is present, may occur between $\frac{3}{8}$ in. and $\frac{3}{16}$ in. when fine and coarse aggregates are obtained separately. An account of naturally-occurring aggregate with a similar grading is given by I. E. Burks [2].

[1] How to Make Good Concrete. Concrete Publications, Ltd. London.
[2] Concreting Methods at Chute-à-Caron Dam. Proc. Am. Conc. Inst., Vol. XXVI (1930), p. 330.

Mixtures with a Cement Content of 520 lb. per Cubic Yard.

Grading No. 1 of *Fig.* 1 was continuous and was used as the control for mixtures with this proportion of cement. The cement-aggregate ratio was 0·16 by weight, corresponding roughly to 1 cwt. of cement to 6 cu. ft. of mixed aggregate. In each of the series of tests A, B, C, E, F, the grading of the sand (0 to $\frac{3}{16}$ in.) was the same as in the control grading, but the percentage of sand was varied. The grading of the sand is shown in *Fig.* 1 (No. S_1). The grading



The numbers refer to the gradings in Fig. 1. ● refers to the left-hand scale; + refers to the right-hand scale. W = workable; S = tendency to segregation; O = oversanded; V = very; Ex. = excellent workability; St. = stony; 29, etc. = time in seconds for initial compaction.

Fig. 2.—Properties of Mixtures Containing 520 lb. of Cement per Cubic Yard.

having 42 per cent. of sand (No. 2, *Fig.* 1) is termed a low-gap grading, that with 52·5 per cent. a middle gap, and that with 63 per cent. a high gap (*Fig.* 1). The results of the more interesting experiments are shown in *Fig.* 2.

SERIES A. CONSTANT WATER-CEMENT RATIO OF 0·6.—The percentage of sand in the aggregate was varied with the same sand grading. With this water-cement ratio the control grading produced a very good concrete. The mixture with a gap-grading and 44·6 per cent. of sand had the best workability, freedom

from segregation, and density. It was more workable and weighed 1 lb. per cubic foot more than concrete made with the control grading.

SERIES B. CONSTANT SLUMP.—The same gradings as in Series A were used, each mixture having just sufficient water to give a slump of about 1¾ in. In series A and B less than 44·5 per cent. of sand led to segregation of the coarse aggregate, while more than 47 per cent. resulted in low density. The most workable concrete of this series had 44·6 per cent. of sand (No. 3, *Fig.* 1); it required 3½ per cent. less water than the control mixture for the same slump, and had greater density.

SERIES C. NEAR GAP. CONSTANT SLUMP (1¾ in.).—An attempt was made to reduce the tendency to segregation in the gap-graded mixture with 42 per cent. of sand by adding 5 per cent. of ⅜-in. to $\frac{3}{16}$-in. material and reducing the amount of ¾-in. to ⅜-in. material by the same amount. The resulting concrete was workable and not liable to segregate, as was the mixture with this proportion of sand in series B. The near-gap grading required 1·5 per cent. less water than that used in the control mixture.

SERIES D. VARIED SAND-GRADINGS. CONSTANT SLUMP (1¾ in.).—The sand grading was varied in the mixtures in which segregation occurred or which contained excess sand in series B, using the r-method and using values of r between 1·4 and 0·9[1]. The r-value is the ratio of the weight of material between each pair of consecutive sieves of the standard series to that between the next lower pair. Gradings other than those conforming to this system were also tried.

Middle-Gap Grading.—Various sands coarser than the control sand were tried to counteract the effects of the high proportion of sand. Gradings with 52·5 per cent. of sands S_2 and S_3 (*Fig.* 1) made concrete weighing about 148 lb. per cubic foot compared with 146 lb. per cubic foot for the middle-gap mixture of series B in which the control sand was used. These mixtures required about the same water content as the control mixture (No. 1, *Fig.* 1) for the same slump; with this percentage of sand a continuous grading would require more water.

Low-Gap Gradings.—Sands finer than the control sand and with r equal to 0·95 and 0·9 were tried to counteract the tendency to segregation. Mixtures with 42 per cent. of sands S_4 ($r = 0{\cdot}95$) and S_5 ($r = 0{\cdot}9$) gave satisfactory concrete with about 5 per cent. less water than the control mixture with the same slump.

SERIES E. CONSTANT WATER-CEMENT RATIO. VIBRATED CONCRETE.—A commercial vibrating table with a frequency of 3000 cycles per minute was used. Each batch was vibrated in a ½ cu. ft. bucket clamped to the table. Experience had shown that if the top surface became smooth and moist in about 30 seconds the compacted concrete would be satisfactory. This condition was called " initial compaction " and its formation in about 30 seconds was used as a criterion of ease of compaction. Using a water-cement ratio of 0·427 (needed by the control mixture for an initial compaction time of 30 seconds) with the various gap-gradings, only the concrete with 42 per cent. of sand was satisfactory. Mixtures with more sand had low densities.

SERIES F. VIBRATED CONCRETE WITH CONSTANT TIME OF VIBRATION.—In this series the proportion of water was that required to give an initial compaction time of about 30 seconds. The control mixture produced a denser con-

[1] The Grading of Aggregates and Workability of Concrete. By Glanville, Collins, and Matthews. Road Research Technical Paper No. 5. D.S.I.R.

crete than the low-gap grading. This is the reverse of what happened with the plastic concrete. Concretes with higher percentages of sand were unsatisfactory because of low densities. A mixture with 37 per cent. of sand had a higher density than the control mixture, and for 26 seconds' initial compaction time required 12·9 per cent. less water. This mixture had less sand than the nominal "low-gap" mixture. No segregation was noticed under laboratory conditions, but trouble might be experienced in practice with this amount of coarse aggregate.

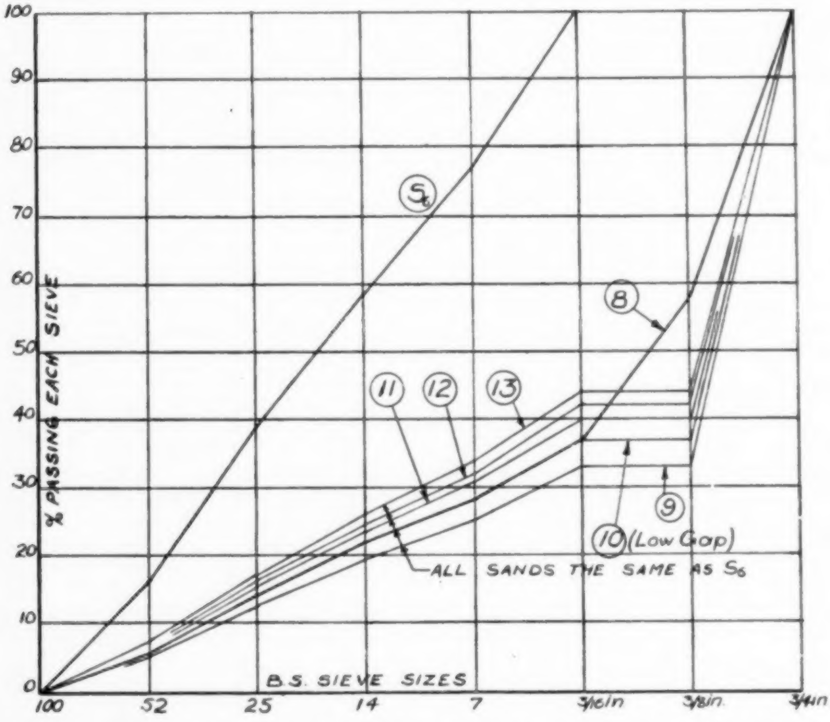


Fig. 3.—Gradings for Mixtures Containing 610 lb. of Cement per Cubic Yard.

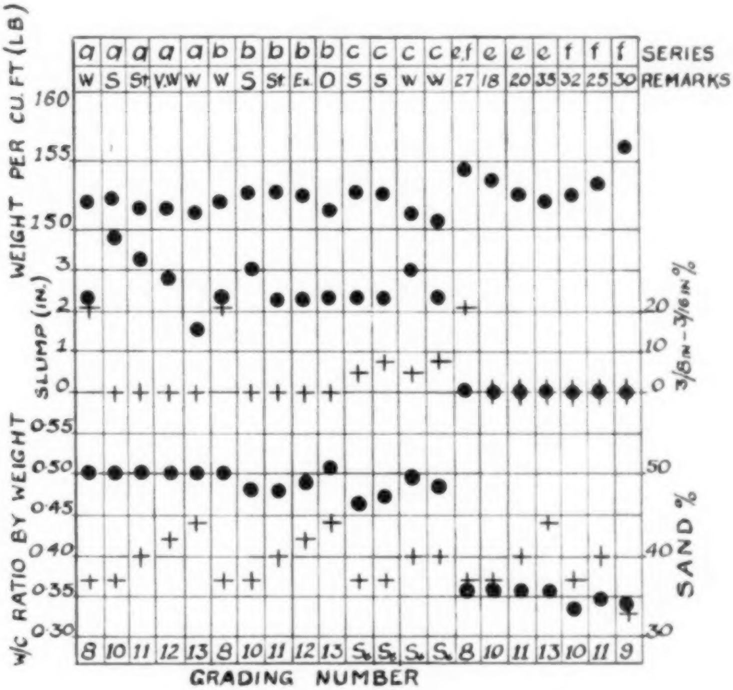
Series E and F showed that for vibrated gap-graded concrete containing 112 lb. of cement to 6 cu. ft. of aggregate there should be between 37 per cent. and 42 per cent. of sand, depending on the shape of the coarse aggregate.

Mixtures with a Cement Content of 610 lb. per Cubic Yard.

The gradings were similar to those described but a different continuous control grading (No. 8, *Fig.* 3) was used which contained less sand and more ¾-in. to ⅜-in. material than the previous control grading. The low-gap grading of this series contained 37 per cent. of sand. The grading of the sand in all the mixtures was the same as that in the control mixture and is shown in *Fig.* 3, No. S_8. The cement-aggregate ratio was 0·186 by weight, corresponding to 1 cwt. of cement to 5 cu. ft. of mixed aggregate. Results of some of the tests are shown in *Fig.* 4.

SERIES a. CONSTANT WATER-CEMENT RATIO OF 0·5.—The continuously-graded mixture produced a good workable concrete which was a little stony. Gap-graded mixtures with less than 43 per cent. of sand were wetter than the control mixture. The most workable concrete had 42 per cent. of sand (No. 12, *Fig.* 3) but it was not quite as dense as that made with the control mixture.

SERIES b. CONSTANT SLUMP.—The slump was $2\frac{1}{4}$ in. which was that of the control concrete having a water-cement ratio of 0·5. It was found that satisfactory concrete could be made with a sand content of between 40 per cent.



[See notes under Fig. 2. Gradings refer to Fig. 3.]

Fig. 4.—Properties of Mixtures Containing 610 lb. of Cement per Cubic Yard.

and 44 per cent. The concrete with 42 per cent. of sand (No. 12, *Fig.* 3) was excellent, requiring 2 per cent. less water than the control mixture and having a greater density; also it was more easily compacted.

SERIES c. NEAR-GAP GRADINGS. CONSTANT SLUMP ($2\frac{1}{4}$ in.).—The $\frac{3}{8}$-in. to $\frac{3}{16}$-in. material necessary to prevent segregation in the low-gap grading produced a continuous grading. Concrete containing 40 per cent. of sand gave excellent results and required less water than the control mixture when 5 per cent. to 8 per cent. of $\frac{3}{8}$-in. to $\frac{3}{16}$-in. material was added. Mixtures C and c showed that the addition of a small amount of $\frac{3}{8}$-in. to $\frac{3}{16}$-in. material may reduce the tendency to segregation in some gap-graded mixtures or may improve the workability of stony mixtures.

SERIES e. CONSTANT WATER-CEMENT RATIO OF 0·356. VIBRATED CONCRETE.—The control mixture required a water-cement ratio of 0·356 for 30 seconds' initial compaction time. Using this water-cement ratio with the gap gradings, concretes with 37 per cent. to 40 per cent. sand reached initial compaction in 20 seconds or less. Concretes with higher percentages of sand were oversanded and had lower densities.

SERIES f. VIBRATED CONCRETE OF CONSTANT TIME OF VIBRATION.—Mixtures with 37 per cent. to 40 per cent. of sand required less water than the control mixture but their densities were lower, as in Series F. Mixtures suitable for vibration were obtained under laboratory conditions with 33 per cent. to 40 per cent. of sand. The mixture with the lower sand content had the greatest density (156 lb. per cubic foot) recorded in the investigation. The high proportion of single-sized stone might, however, cause trouble in placing the concrete in practice.

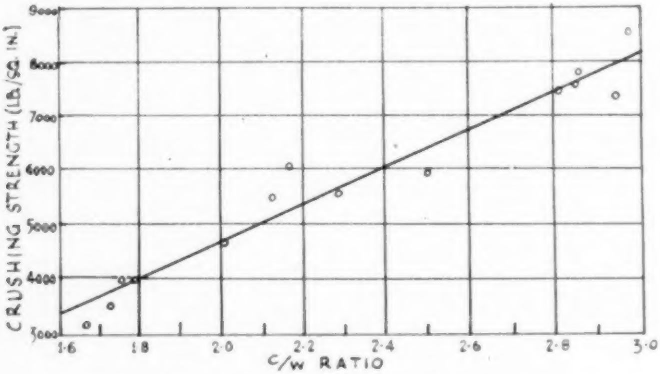


Fig. 5.—Strengths of 6-in. Cubes at Seven Days.

Strength Tests.

The results of crushing tests at seven days on 6-in. cubes made with the concretes described are shown in *Fig.* 5. The strengths are plotted against the corresponding cement-water ratios rather than the water-cement ratios as this gives an approximately straight-line relationship.

Conclusions.—The tests show that for the $\frac{3}{4}$-in. aggregate used certain gap-gradings omitting $\frac{3}{8}$-in. to $\frac{3}{16}$-in. material may produce a concrete with better workability, density, and water requirement than a continuous grading. They show that care must be taken in choosing the proportion of sand, as a wrong choice may result in concrete liable to segregation due to an excess of coarse aggregate, or concrete which has a low density and high water requirement due to an excess of sand.

The experiments were carried out in the laboratory of the Department of Civil Engineering at University College, Cork. The author expresses his thanks to Professor H. N. Walsh for his guidance and help in the experimental work and in the preparation of this paper.

A Large Gasholder Tank.

The gasholder tank described is at the Stapleton Road, Bristol, works of the South-Western Gas Board. The gasholder is to be a four-lift, spirally-guided, all-welded, water-sealed structure. The tank is 232 ft. 2 in. internal diameter and 42 ft. 10 in. deep inside. The general details of the tank are shown in *Fig.* 1. Two valve-chambers were constructed monolithically with the tank. At the centre of the tank, support is provided for the post to carry a load of 210 tons.

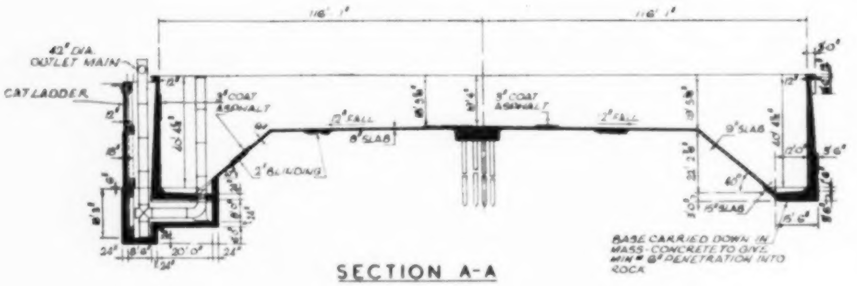
The site rises steeply, and is partly on an old clay pit about 50 ft. deep which had been filled with ashes and town's refuse. Bores indicated that the filling existed to a depth of about 50 ft. below ground level and that there was a very hard compressed Keuper Marl rock formation below. The rock was nearer the surface on the northern side of the clay pits, but it was not expected that this stratum would be encountered within the area of excavation for the tank. Borings gave no indication of any standing water-table, and chemical analyses gave no indication of the presence of chemicals that would harm concrete. Subsequent experience, however, proved otherwise, and the borings actually gave a very unrepresentative picture of actual conditions.

A bearing capacity of about 3½ tons a square foot was required, and this could not be provided by the filling. The height of the tank and the fully inflated holder was about 216 ft. and, as a line drawn at 45 deg. to the horizontal from the ground-floor window of the nearest house had to pass over the top of the inflated tank, it was necessary to construct the tank in the ground so that the height above ground level was 175 ft. As the average depth to the marl was between 50 ft. and 55 ft. the top of the tank was at + 9 ft. 6 in., the original average ground level being + 18 ft. For about half of its perimeter the base bears directly on rock ; for the remainder, plain concrete was placed below the base to a maximum thickness of 3 ft. The excavation was done in open-cut, the batters being generally 1 in 1, except that where the base was outside the old clay workings a much steeper gradient was permissible. The concrete was distributed by pump because of the large amount required and the difficulty of access.

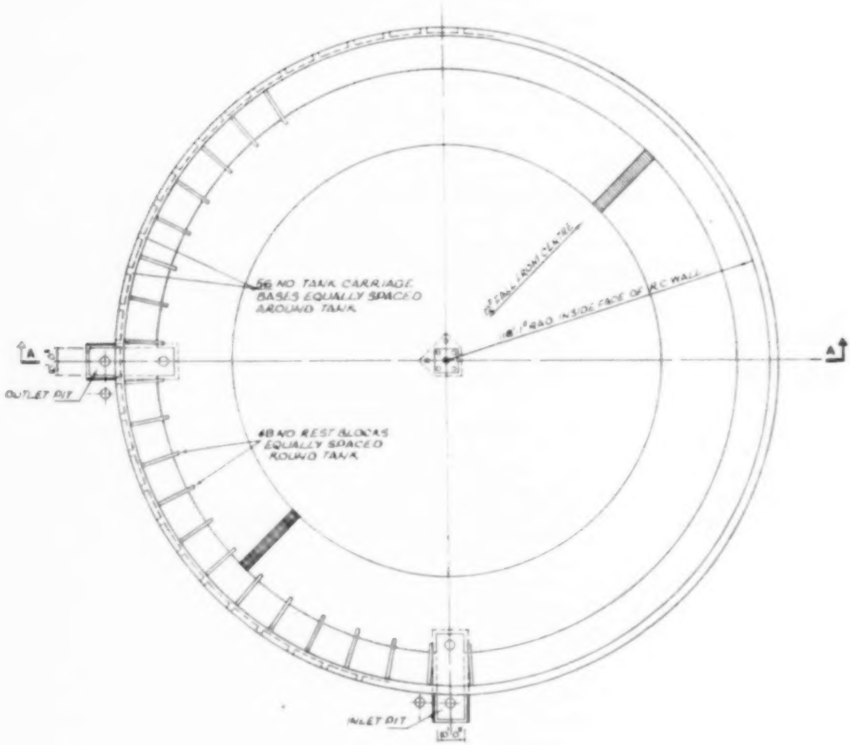
Design.

Three methods of designing the wall were considered, namely, (i) With a " free " joint at the base, the whole of the hydraulic pressure from within the tank being resisted by ring tension ; (ii) The wall to be monolithic with the base slab, the hydraulic pressure being resisted partially by ring tension and partially by the wall acting as a cantilever ; and (iii) A prestressed concrete wall. Preliminary calculations indicated that the thicknesses of the base of the wall for these methods would be (i) 5 ft., (ii) 3 ft. 6 in., and (iii) 2 ft. plus the cover to the prestressing wires.

It was considered that a " free " joint at the base gave no advantage other than simplifying the calculations, and no further consideration was given to this type of construction. The saving in concrete and steel in a prestressed tank (iii) compared with a monolithic structure (ii) appeared to indicate that a fairly substantial saving in cost could be made by adopting a prestressed tank, but closer analysis showed that this was not the case. The most practical method of distributing concrete was by pump and it was considered that it would not be possible on this site to guarantee a strength at 28 days of 6500 lb. per square inch by pumping. In addition, a prestressed tank would not normally be asphalted internally, and it would be necessary to test the tank before backfilling. A prestressed tank would therefore have to be designed to resist full hydrostatic pressure with no relief from the active earth pressure, whereas in the case of a monolithic structure (ii) the inside of the tank would be asphalted and back-filling would be completed before the tank was filled with water so that allowance could be made for the effect of the active earth pressure. The maximum ring tension at the base of a prestressed tank would therefore be substantially greater, and, to obtain the precompression, with a sliding joint at the base, a total area of 2·75 sq. in. of 8-gauge wire per foot would be required. This would need 170 wires per foot and necessitate placing the wires in three or



SECTION A-A



PLAN OF TANK

Fig. 1.

four layers at the base of the tank (the number of layers decreasing at intervals to the top), each layer being protected by a layer of gunite before placing the subsequent layers of wire. Furthermore, due to conditions on the site, it would have been uneconomical to handle shutters of sufficient strength to enable the full height of the wall to be concreted as a continuous operation in segments, and therefore some form of vertical prestressing would be desirable. Allowing for these factors, it was found that a prestressed tank showed no saving in cost compared with a reinforced concrete tank, and a reinforced concrete tank with the walls monolithic with the base (ii) was built.

The design followed generally the Recommendations for Liquid Retaining Structures of the Institution of Civil Engineers, but slight increases in the stress in the steel and of the net tensile stress in the concrete due to direct forces were allowed, because (a) the tank would be founded on rock, (b) no difficulty was expected in producing concrete with a minimum strength at 28 days of 3500 lb. per square inch, and (c) vibrators would be used, and care taken to ensure that shrinkage was small.

The maximum stresses used were: Steel, 14,000 lb. per square inch in direct tension and bending on the water-face and 18,000 lb. on the remote face; concrete in compression, 850 lb. per square inch due to bending and 600 lb. due to direct forces; concrete in tension, 250 lb. per square inch due to bending, and 200 lb. due to direct forces calculated on the composite section.

The wall was designed in accordance with the method of Dr. Reissner. Due to the monolithic construction the deflection of the wall under hydrostatic pressure is restricted at the base and bending moments are produced, the wall tending to act as a propped cantilever due to ring tension higher up. Progressing upwards from the base, the bending moment, which is negative at the base, reduces rapidly to a point of contraflexure, a positive moment then developing and being greatest at about the point of maximum ring tension and producing tensile stresses in the outer face. The ring tension increases fairly uniformly from the top to a maximum at a depth depending upon the ratios of the height of the wall to the diameter of the tank and

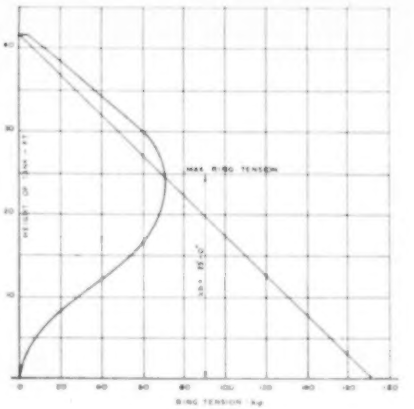


Fig. 2.—Variation of Ring Tension with Height of Wall.

the thickness of the wall. The maximum ring tension, its position, and the maximum negative moment at the base may be expressed in terms of parameters obtained from Dr. Reissner's analysis, allowance being made for the taper of the wall. The maximum ring tension is 25 ft. from the base (*Fig.* 2) and, allowing a relief from the active ground pressure of 27 lb. per foot of depth, the maximum ring tension required 4·9 sq. in. of reinforcement per foot of height. The tensile stress in the composite section was then 193 lb. per square inch or 202 lb. per square inch, dependent upon whether the modular ratio is assumed to be 15 or 12. From this point the direct tension decreases at a reasonably uniform rate to the top and bottom of the wall.

The bending-moment diagrams for the combined effects of water pressure and earth pressure, and for earth pressure alone, are given in *Fig.* 3. Steel is provided vertically to resist these moments together with certain additional moments from the holder, which are referred to later. Sections showing the reinforcement are given in *Fig.* 4.

Around the top of the tank are 56 anchor-blocks which carry the spiral guides of the holder. When the holder is inflated, wind will cause horizontal shearing forces to act on these blocks, together with downward forces on the leeward side and upward forces on the windward side. The point of application of the vertical forces is 15 in. inside the tank. Due to

the thickness of the wall it was assumed that the moments produced by the vertical forces from the guides would be distributed over a length of wall equal to the distance between the anchor-blocks. As the tank is fully buried in the ground, no special measures were taken to resist the horizontal shearing forces from the spiral guides, as it was considered that these forces would be resisted by the passive ground pressure. In addition, the balcony beam at the top of the tank was reinforced to form a stiffening rib.

At the base of the tank, 48 rest-blocks are spaced equally around the circumference to carry the steelwork of the holder when it is partially or wholly deflated. The maximum load per block is 28 tons. The slab is designed to spread this load more or less uniformly over the base. The maximum ground pressure assuming the base to be 14 ft. wide and to cantilever from the wall, was 7840 lb. per square foot, which was satisfactory as the tank was founded on Keuper Marl. Adding the effects of the reaction of the ground and the downward loads gave the bending moments to be resisted by the footing.

As a precaution against unequal settlement due to the large difference between the load at the base of the tank and that at the valve-pits, additional shear bars are placed in the walls of the valve tunnel.

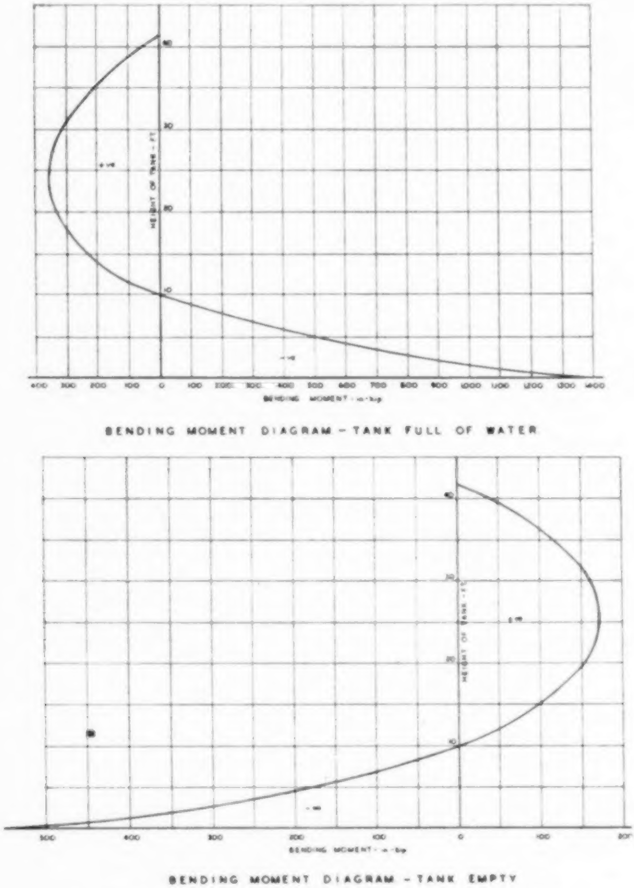


Fig. 3.—Bending Moments on the Tank Walls.

Consideration was given to welding the circumferential reinforcement in the wall of the tank. This would have saved 12 tons of steel. Due to other commitments, however, the contractor was not able to provide welders and the cost of employing a sub-contractor was, in this case, greater than the saving in reinforcement.

Foundation of Centre-Post.

The original design of the centre-post foundation utilised a circular reinforced concrete caisson to transmit the load of 210 tons to the Keuper Marl, which was estimated to be 28 ft. below "dumpling" level. However, due to the presence of water in the subsoil, and to the fact that piling equipment and material were available from an adjoining site, it was decided to drive five concrete piles of 1 ft. 6 in. diameter and form a pile cap 3 ft. thick to carry the load. The estimated length of the piles was 24 ft., but all the piles reached a set at 13 ft. below dumpling level. In view of the difference between the estimated length of the piles and the actual length it was decided to drive four test piles to ascertain the rock formation at a distance of 20 ft. from the centre of the pile cap. Those on the north, south and east axes indicated a reasonably level rock stratum. The pile driven on the west side reached a set at 25 ft. 6 in. below the dumpling, and a pile driven 10 ft. from the centre of the cap on the western axis reached a set 21 ft. below dumpling level. In view of the doubtful bearing strength of the structural pile driven on the western axis two more piles were driven 4 ft. from this pile on the N.W. and S.W. axes of the cap. These reached a set at 13 ft. 6 in. and 19 ft. respectively below dumpling level, and were later incorporated as load-bearing piles in the cap.

"Dumpling" Slab.

To simplify the erection of the steelwork for the holder, the dumpling slab was required to be as large as possible. The original specification required the slope from the main channel, accommodating the shell of the holder, to the central dumpling to be at 60 deg. to the horizontal. At the request of the contractor, this was reduced to 40 deg. to avoid having to use an outer shutter. No great difficulty was found in placing concrete at this slope (*Fig.* 5), but where practicable it would no doubt be preferable to have a slope of 35 deg. or less. The main dumpling slab is 8 in. thick and is reinforced with a light mesh in each face. Shrinkage was reduced by careful selection of the sequence of concreting the panels of the slab.

The specification required that the work should conform to the following dimensional tolerances: Diameter of the tank, $\pm \frac{1}{2}$ in.; Level of the rest blocks, $- \frac{1}{8}$ in.; Plumb of wall, $\pm \frac{1}{4}$ in.; Positions of anchor blocks, $\pm \frac{1}{8}$ in.; Distance between anchor blocks and rest blocks, $\pm \frac{1}{2}$ in.

Excavation.

The material excavated down to dumpling level was 76,000 cu. yd.; standing water was found 4 ft. below dumpling level. It was thought that this was surface water which had collected in the clay pit, and two shafts were sunk 6 ft. into the foundation rock to form collection chambers from which to pump the water. As it was impracticable to use a well-point system, the water, the source of which was later found to be springs coming through the rock surface, was taken by channels to sumps on the south side outside the perimeter of the tank and then pumped to a settling-tank at ground level, from which it flowed by gravity into the sewer. The water-level was lowered 22 ft. to the rock but a continuous flow of about 10,000 gallons per hour was maintained and this rate of pumping continued during the construction of the tank.

During the excavation of the dewatering shafts it was noted that the water flowed into the pit in surges which were accompanied by obnoxious smells. This was due to the release of pockets of gas from the decayed vegetation in the surrounding "fill" and it was necessary to pump air to the men excavating the shafts.

While excavating the trench for the retaining wall on the northern perimeter it was found that the presumed boundary of the rock which had been traced by a survey actually extended the full width of the trench and commenced at a level just below the dumpling. This rock had to be removed by blasting which was carried out on an 8-ft. face in two steps,

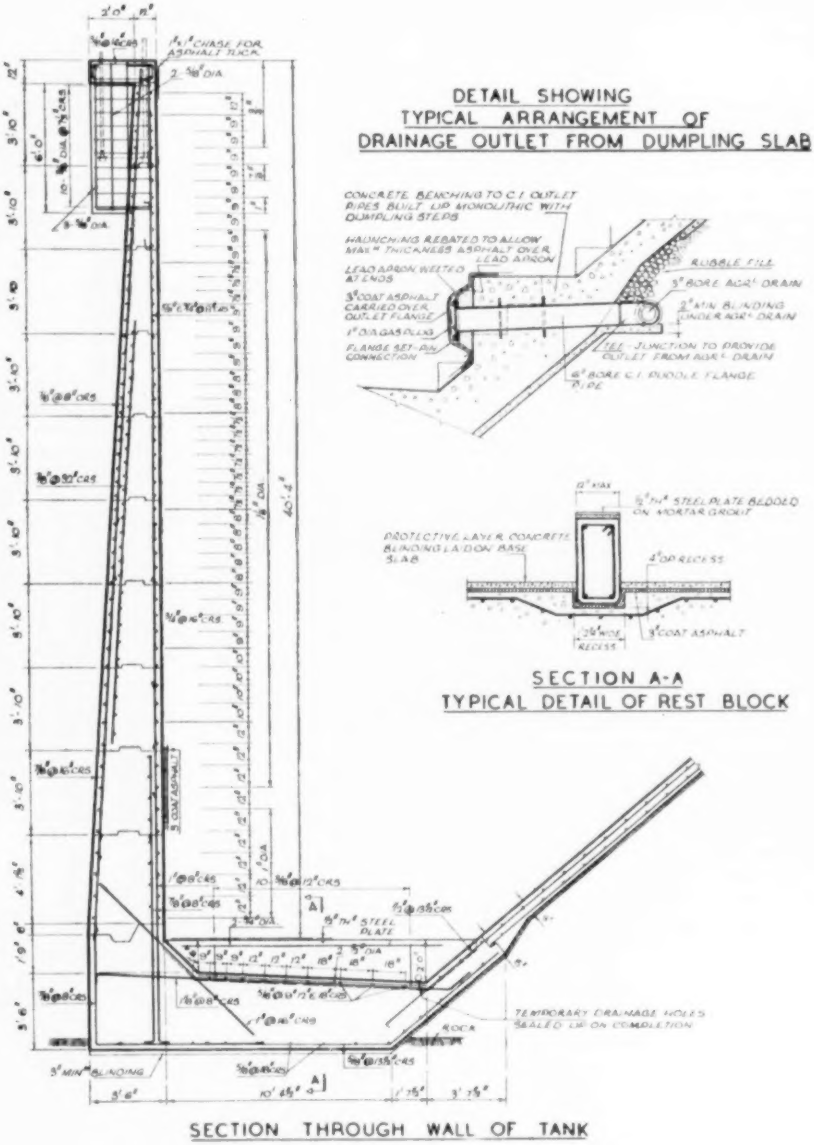


Fig. 4.—Details of Reinforcement in the Wall.

the centre being worked in advance of the sides. Holes were drilled to a depth of about 4 ft. and each charged with 1½ lb. of plastic explosive. It was usual to fire a series of eight charges with a short interval between firing each charge, using cordite fuse and amatol detonators. No blanketing mat was necessary for these operations.

Protection against Sulphates.

In the S.E. area of the site a white precipitate formed on the surface of the blinding concrete, and on the surface of the ground after excavation discoloration appeared accompanied by obnoxious gases. The sulphate content of samples of earth and water showed a marked increase over that of the samples previously taken, and in view of the high sulphate content, (a) All concrete, including the blinding layers, had a minimum cement content of 1 : 6 ; (b) Where possible, all concrete in contact with the black earth filling or sub-soil water was given two coats of bitumen applied hot ; (c) At the junction of the base of the retaining wall and the rock a fillet of high-alumina cement concrete was laid to form a seal with a lap of 1 ft. over the bituminous membrane on the wall, and the joint sealed with two coats of bitumen, as it was not possible to apply bitumen to this concrete due to the presence of water ; (d) The walls of the valve-pits which were exposed to the atmosphere were lined with asphalt on the faces in contact with the earth filling. The probable reason for the increase in sulphate content was oxidisation of some of the filled material following excavation.

Due to the position and level of the main-line railway in relation to the bottom of the excavation, and the presence of sub-soil water, it was necessary to sheet-pile a length of 120 ft. of the trench. The general level of the rock was 3 ft. below the base of the retaining wall and a concrete filling (1 : 2 : 4 mixture) was laid to make up this variation, and also to fill the sump of an old pumping chamber into which spring-water was flowing through fissures in the rock. The setting of the concrete first placed was delayed, and a sample of the spring-water was found to contain bicarbonates which reacted with the $Ca(OH)_2$ in the cement. The water was therefore kept away from the concrete until the final set took place, and additional sumps

Fig. 5. Construction of the Dumpling Slab.

and drainage channels were constructed to remove this water.

Construction.

The base of the retaining wall was constructed as soon as possible behind the excavation of the ground. The first lift of the wall, for which a special set of shutters was made, was also completed for the full circumference and followed closely upon the construction of the base. It was possible at an early stage to back-fill to a height of 6 ft. The main wall was built in halves, in each case to the full height. This led to economical use of the shutters and reduced the amount of scaffolding required.

A 4-in. concrete pump driven by a 21-h.p. motor and an electrically-driven 21/14 closed-drum mixer with a raised discharge were installed. All materials were weigh-batched with a trolley batcher placed on an embankment so that lorries could discharge into chutes feeding the storage bins. All concrete was vibrated with electrically-driven immersion-type vibrators and hammers applied to the surface shuttering. The concrete was required to have a minimum strength of 3500 lb. per square inch at 28 days and it was found that a mixture of 448 lb. of limestone, 336 lb. sand, and 112 lb. of cement gave the required strength and workability. The slump was usually between 1 in. and 3 in. depending on the part of the work being cast and the arrangement of the pipe-line.

The pipe-lines were inclined downwards, which is normally to be avoided when using a concrete pump. The concreting of the upper lifts of the wall, which were above dumpling level, needed particular care as the pipe-line inclined downwards from the mixer to the dumpling, and then rose to the point of discharge, thus forming a " trap " in the pipe-line. In order to overcome this difficulty a loosely-packed " paper dolly " was placed in the pipe and a mixture of sand and cement was pumped through the pipe before the concrete. The mortar lubricated the internal lining of the pipe and the " dolly " prevented separation of the mortar on the downward inclination, thus preventing air being sucked into the pipe and avoiding air locks and blockages. From the pump to the rim of the excavation a slight rise in the pipe-line was maintained in order to keep the face of the piston fully charged with concrete. It was usual to terminate the pipe 10 ft. above the placing position and to distribute the concrete within the shutters by means of flexible " elephants' trunks ". The concreting of the base was divided into 28 sections, each containing $67\frac{1}{2}$ cu. yd. of concrete, this being assumed to be a day's work (the actual concreting time being about eight hours). The walls were cast in 4-ft. lifts; in the lower lifts of the wall 250 cu. yd. of concrete were placed in a working week of six days.

The sides of the dumpling sloping at 40 deg. were concreted in sections about 12 ft. wide without a face shutter.

SHUTTERING.—There was about 7000 sq. yd. of shuttering to the wall of which 3400 sq. yd. had a sloping face. The shuttering comprised timber panels 4 ft. high with 8-ft. soldiers on curved steel walings. Each panel had a length of $\frac{1}{56}$ of the circumference, the internal shutters giving a working clearance of $\frac{1}{4}$-in. in the length, and the external shutters, which were to a sloping face, to $\frac{1}{56}$ of the least circumference. On the lower lifts the gap was made up by filler pieces which varied in width for each lift. In each lift, square nuts, with an anchor and anti-turning device of two straight lengths of square rod 4 in. long welded to each nut, were cast in the wall at a distance of 4 in. from the face. The shutters were hoisted by means of a fixed chain-block secured to the scaffold. Twenty-eight panels, covering half the circumference of the wall, were provided.

On the ninth and tenth lifts gaps were left in the concrete at positions where the groups of holding-down bolts for the bases of the gasholder roller-carriage were to be set. Later the bolts were fixed, using templates, and concreted in. The general contractors, who also designed the reinforced concrete, are Messrs. A. Monk & Co., Ltd., the contractor for the holder is the Oxley Engineering Co., Ltd., and the asphalt work was carried out by Highways Construction, Ltd.

Precast Buildings in Eire.

PRECAST concrete members have been used for the framework of new tarehouses erected at three of the factories of the Irish Sugar Co., Ltd. (Tarehouses are buildings in which the tare and sugar content of each load of sugar-beet are measured.) The design of the frames is unusual as the roof members are pin-jointed at the tops of the columns, which therefore act as cantilevers to resist the horizontal thrust from the members. The frames are shown in *Fig.* 1.

Precast Members.

The frames (*Fig.* 2) are at 13 ft. 4 in. centres for a span of 29 ft. 5 in., the roof having a rise of 10 ft. The cross section of the rafters is 9½ in. by 6 in. and they are haunched at the apex. Projections, 3½ in. by 3 in., are provided as seats for the purlins, which are also of precast concrete.

The columns (*Fig.* 4) are tee-section, 14 ft. 6 in. long, with a recess formed at the top in which the roof members are bolted. The purlins are 13 ft. 3¾ in. long by 6 in. by 3 in. and are connected to the rafters by ⅝-in. diameter bolts passing through ¼-in. steel plates. The foundations for the columns, 5 ft. 6 in. long by 3 ft. wide, were cast three weeks before the erection of the columns, which were placed eccentrically on the foundations. The main reinforcement in the columns projects at the bottom (*Fig.* 4) and is welded to the same number of bars pro-

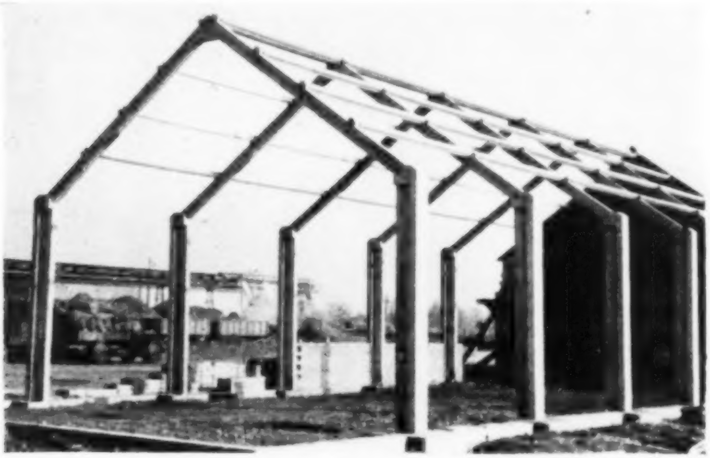


Fig. 1. Partially-erected Frame.

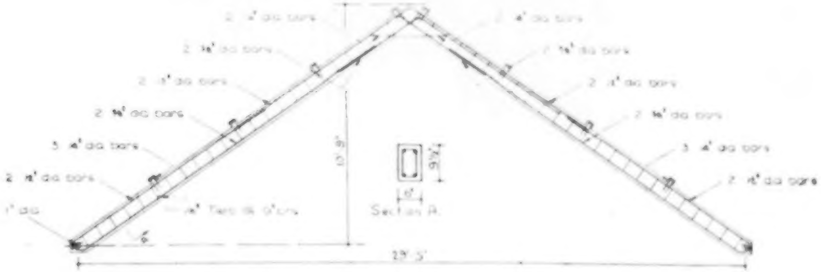


Fig. 2. Details of Roof Frame.

jecting from the in-situ foundation (*Fig.* 3). This reinforcement is then encased with a 1 : 2 mixture of mortar, thereby ensuring continuity between the columns and the foundations.

The roofs are covered with asbestos-cement sheets fastened to the purlins by hook-bolts. The external walls are of 9-in. concrete blocks.

The concrete for all the members is a 1 : 1½ : 3 mixture with aggregate of ⅜ in. maximum size for the roof members and ¾ in. for the columns. The concrete was vibrated by portable external vibrators. The water-cement ratio was 0·43 and crushing strengths of about 6500 lb. per square inch were obtained on cubes at an age of 14 days. The roof members and the columns weigh about 1 ton each, and the purlins 2 cwt. The reinforcement was welded in cages before it was placed in the moulds.

Erection.

The members were cast at a works and transported to the sites by lorries. They were erected by a 10/RB crane. The order of erection was first to place the columns on the foundations and temporarily shore them. The roof frames were then placed in position and bolted to the columns, after which the eaves purlins were bolted to the frames. The columns were then aligned and the reinforcement projecting from the base of the columns was welded to that projecting from the foundation, after which the joint was concreted. The other purlins were then placed followed by the asbestos-cement sheets.

Holes were formed in the rafters from which 4-in. by 2-in. timber joists could be supported to carry ceiling joists to which were nailed insulating boards. The finished ceiling therefore conforms in shape to the outline of the roof frames.

Fig. 3.—Joint between the Reinforcement of the Columns and the Bases.

Design.

The members were designed by Mr. P. J. Carroll, M.E., A.M.I.C.E., engineer of the Irish Sugar Co., Ltd., and follow the principles set out by him in "The Factor of Safety as Applied to Reinforced Concrete Design," published in the November, 1951, number of the Journal of the Institution of Civil Engineers. In this paper it is stated that, provided deflection is

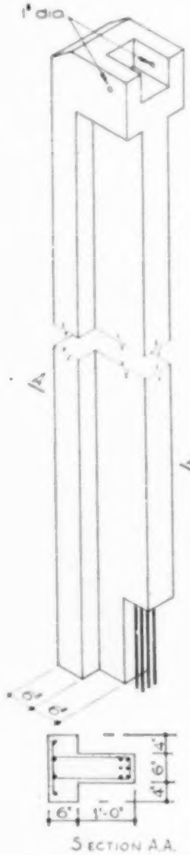


Fig. 4.—Details of the Columns.

not of great importance, the members need be designed to carry only a proportion of the dead load (in this case 68 per cent.) together with the full imposed load. This results in a considerable reduction of the sizes of the members in cases where the ratio of the dead load to the imposed load is large.

Thin Concrete Walls and Roofs.

GUNITE ON BAMBOO AND WIRE-MESH REINFORCEMENT.

We have received the following notes on the use of thin concrete walls and roofs in India from Mr. M. R. Venkataram, chief engineer of the Western Railway of India. A similar form of construction was described in this journal for October, 1952.

In the year 1948, in the area of the Western Railway of India, small dwellings were constructed of gunite, 2 in. thick, on thin bamboo reinforcement tacked on in diamond formation to wooden posts and walings to form the walls; the roof was of asbestos-cement sheets on wooden rafters. An office building and several other buildings of the same construction have withstood five monsoons with rainfalls of over 80 in. and including cyclones. They are still in excellent condition although hair cracks have appeared in the concrete at the wooden posts due to lack of adhesion. It is thought that these cracks could have been prevented if steel nails had been partly driven into the sides of the posts and bonded in the gunite.

Other buildings have been constructed in which bamboo reinforcement was replaced by 17-S.W.G. steel wire or by 2-in. wire mesh tied on to frames of ⅜-in. diameter bars, all of which were embedded in 2 in. of gunite to form the wall panels. The vertical bars at the edges of the frames formed the reinforcement of the columns. These buildings have withstood three monsoons with over 100 in. of rainfall; at one time the rainfall was 30 in. in 24 hours.

To avoid having to take the gunite equipment to each site, and also to reduce shrinkage after erection, the members were precast. Single and multiple-story buildings with flat roofs have been con-

Fig. 1.—Residential Flats at Bombay, India.

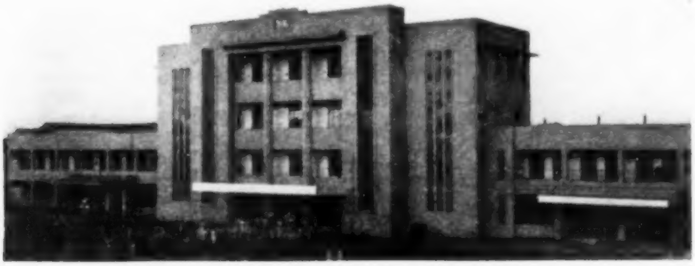


Fig. 2.—Western Railway Station, Surat, India.

structed with precast members. The wall panels are 2 in. thick and the columns were either cast in a single length for each story or made of hollow members each 1 ft. deep. The columns have grooves in which the wall slabs were placed, bedded in mortar, and grouted. The beams were also precast. The roofs comprise precast slabs from 2½ in. to 3 in. thick, laid on ledges formed in the beams, and with bars projecting into 1½ in. to 2 in. of concrete laid in situ. *Fig.* 1 shows a block of flats and *Fig.* 2 the station buildings at Surat constructed in this manner. Where climatic conditions are extreme, hollow walls with an air space of 2 in. between the leaves have been found adequate. The slabs are cast on a concrete platform which is oiled, giving a smooth surface to the slabs and obviating the need for internal plaster in the Indian climate.

The concrete mixture was 1 : 4 for the roof and load-bearing members and 1 : 6 for the walls. Tests indicate that both the roof and wall slabs are very strong and have a factor of safety of six. The design stresses adopted were 1000 lb. per square inch compression in the concrete and 18,000 lb. per square inch tension in the steel.

The cost of these houses compared with conventional types of single-story houses is 20 to 25 per cent. less for the single-wall types and about 12 to 13 per cent. less for the hollow-wall types. In the case of multiple-story dwellings the saving compared with conventional construction varies from 10 to 30 per cent. Large station buildings, offices, and warehouses have been built by these methods on this railway.

Mr. M. R. Venkataram, whose address is General Office, Western Railway, Churchgate, Bombay, India, informs us that he will be pleased to send to those interested full information on the methods he has developed (which have not been patented).

Book Reviews.

"Prestressed Concrete." By Kurt Billig. (London: Macmillan & Co., Ltd. 1952. Price 36s.)

THE arrangement of this book is similar to that by the same writer published in 1944. It is divided into three sections, the first dealing with the development of prestressed concrete and describing many of the systems in use, the second describing methods of analysis, and the third giving examples of the design of structures.

The most successful part of the book is the first section in which the most important applications are briefly, but adequately, described. The chapters which follow, under the heading "Design", deal entirely with the stress analysis of a section. There is no development of a method leading logically from the calculation of the bending moment to the dimensioning of a member. Nor, for the cases in which curved cables are used, is a method given for determining quickly the shape of the cable when its maximum eccentricity has been calculated. It is stated that the maximum value of the shearing stress is unaltered whether the beam is prestressed or not, but there is no mention of the fact that, if the beam contains cables curved upwards towards the ends, the shearing force due to the load is reduced by the vertical component of the slope of the cable.

Among the examples are calculations for a transmission-line pole in the form of a tapering Vierendeel girder, and the design of a shell roof in which the slab, $1\frac{3}{4}$ in. thick, is cast with longitudinal grooves, 1-in. deep, in which wires are placed and tensioned and the roof then covered with gunite.

"Dimensionnement Experimental des Constructions." By Manuel Rocha. (Lisbon: Ministry of Public Works. 1952. No price stated.)

THIS is a review, in the French language, of the methods available for the design of structures based upon the behaviour of scale models. Mechanical similarity is considered for two cases, the one in which both model and prototype are of the same material and the other in which the materials differ. The construction of models, methods of applying loads or deflections, and of measuring deformations are discussed in detail and the advantages and disadvantages of various types of extensometers, including electrical resistance strain gauges, are described.

"Architects' Detail Sheets." Edited by Edward D. Mills. (London: Iliffe & Sons, Ltd. 1953. Price 25s.)

THIS book contains 96 scale detail drawings accompanied by photographs of portions of buildings completed during the last few years. The details have been contributed by architects in many countries and are a selection of those published since the year 1948 in "The Architect and Building News." Although the book is primarily of interest to architects the details of reinforced concrete work will be of value to engineers concerned with the structural design of buildings.

"Funções Ortogonais na Resolução de Problemas da Teoria da Elasticidade." By T. van Langendonck. (São Paulo: Portland Cement Association of Brazil. 1952.)

THIS is the first of two volumes of a book in the Portuguese language dealing with the solution of various problems in the theory of elasticity by the use of orthogonal functions. The general theory is introduced in the first chapter and the remainder of the book is devoted to the application of the method to problems of torsion on sections of various shapes including hollow sections and circular shafts of varying diameters.

"Quality Concrete." The Concrete Association of India. (Bombay: The Association. 1951. Price 7/8 rupees.)

IN the first four chapters of this book information is given regarding the manufacture of Portland and other cements, the handling and testing of cement, the types and quality of aggregates, the testing of aggregates, the testing of water for concrete, and the types and uses of admixtures. Various methods are described, suitable for large and small contracts, for proportioning mixtures. The chapter on control of the quality of concrete on the site is comprehensive and includes methods of transporting, placing, compacting and curing concrete as well as methods of sampling and testing. Appendixes include standard methods of making cube and beam tests, details of the effects of various substances on concrete, and notes on the Indian draft specifications for cements.

Generally the specifications and standards are those of the British Standards Institution but the methods of proportioning mixtures and site control are based on U.S.A. practice.

Walls of Blocks and Slabs.

Code of Practice No. 122 (1952) "Walls and Partitions of Blocks and Slabs" has been issued (price 9s.) by the British Standards Institution. The materials dealt with in the code are blocks of clay, concrete, gypsum, anhydrite, and glass, and slabs of woodwool and plasterboard and plaster.

The recommendations for concrete blocks are the same as in British Standards Nos. 492, 728, and 834. In addition the code gives information regarding sound insulation, thermal insulation, fire resistance, the reduction of condensation, and the prevention of pest infestation. The causes of cracks and the precautions to be taken to prevent their formation are outlined.

In the section dealing with work on the site recommendations are made regarding the handling and storage of materials, methods of laying blocks, the provisions to be made for services and fittings, and the treatments commonly used at the heads of load-bearing and non-load-bearing walls and partitions.

A Composite Tee-Beam Floor.

THE system of floor and roof construction shown in *Fig.* 1 may be used for spans of moderate lengths for buildings for office or domestic use. It comprises precast concrete beams with a concrete slab $1\frac{1}{2}$ in. to 2 in. thick cast in situ on a shutter of close-mesh light-weight expanded metal. Bond between the beam and the slab is obtained by stirrups which project through the mesh and are bent over to lie in the slab.

The construction of a floor supported by in-situ columns and main beams is shown in *Figs.* 2 and 3. The main beams are T-shaped, with a flange of the same depth as the precast beams. The ends of the longitudinal bars in the precast beams project and are embedded in the flanges of the main beams. A timber nailing strip, which also forms the bottom of the mould, is attached to the underside of each precast beam to enable the ceiling to be fixed. After the shutters for the main beams are erected the precast beams are placed in position, supported on the shutters at their ends and by a central prop (*Fig.* 2). The main beams are then concreted, and later the expanded metal is placed in position and the slab laid (*Fig.* 3).

Where a floor is supported by walls, the

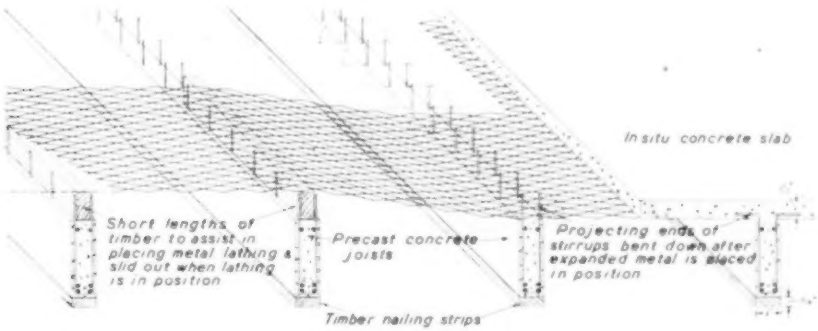


Fig. 1.

Fig. 2.—Precast Beams laid on Shutters of Main Beams.

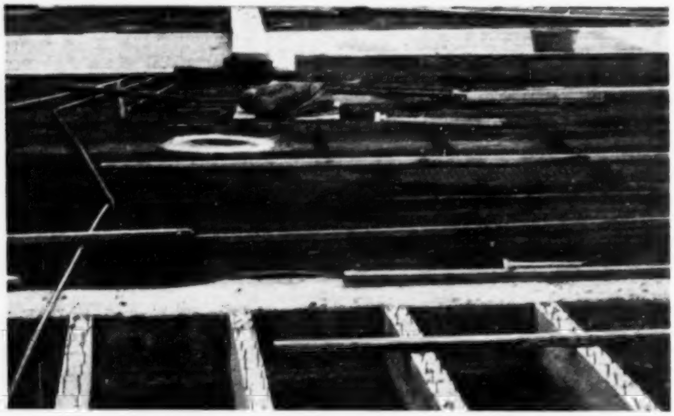


Fig. 3.—Floor Ready for Laying Topping.

arrangement of the precast beams is the same as that for timber joists. The spacing of the precast beams can be varied to suit the dimensions of the floor and the load, but is limited by the type of ceiling to be provided. For a plasterboard ceiling the spacing is usually 1 ft. 6 in. Where light-weight partitions have to be supported, two precast beams may be placed side by side. For spans up to 12 ft. the average cost, exclusive of the ceiling and floor or roof finishes, is 26*s.* per square yard. The system has been designed by the Expanded Metal Co., Ltd.

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Situations Wanted, 3d. a word: minimum 7s. 6d. Situations Vacant, 4d. a word: minimum 10s. Other miscellaneous advertisements, 4d. a word: 10s. minimum. Box number 1s. extra. The engagement of persons answering these advertisements is subject to the Notification of Vacancies Order, 1952.

Advertisements must reach this office by the 23rd of the month preceding publication.

SITUATIONS VACANT.

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(Continued on p. lxvi.)

(*Continued from p.* lxv.)

SITUATION VACANT. A Company specialising in the design of reinforced concrete buildings requires an engineer for site inspection of work carried out by contractors to the Company's designs. Applicants should have held responsible positions as resident engineers or agents for reinforced concrete constructional work. Visits to sites in all parts of the country will be entailed. Some design experience is desirable. Write, giving details of age, experience, and salary required, to Box C.E.236, 191 Gresham House, London, E.C.2.

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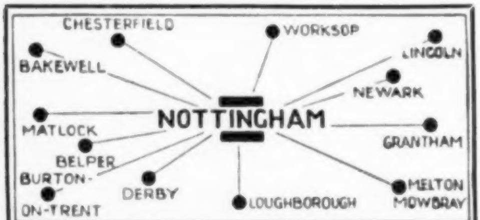

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